Missouri Department of Transportation

Bridge Division

Bridge Design Manual

Section 8.2

Revised 05/04/2000

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8.2.1 Hydraulic Design Criteria

8.2.1.1 Design Frequency

Bridges and culverts are designed to pass the design flood discharge and at the same time meet backwater criteria.

The return period or frequency of occurrence of an event is the average period of time between events equal to or exceeding the given magnitude. The annual probability of occurrence of an event is equal to the reciprocal of the return period. For example, a flood with a return period of 100 years has a 1% chance of occurring in any given year; whereas a flood with a return period of 25 years has a 4% chance of occurring in any given year.

The design frequency, or return period of the design flood, varies by type of construction.

New structures

A 100-year design frequency is to be used for all new structures. The ability of the proposed design to pass other flood flows, including the 500-year flood discharge, should be evaluated to determine potential for significant damage to adjacent properties and the highway facility. If the 500-year discharge is not available, use a value of 1.7 times the 100-year discharge.

New structures designed with an overflow section (low roadway approaches) are to pass the entire design discharge through the bridge opening and still meet backwater criteria. The capacity of the overflow section is ignored.

Widening, rehabilitation, or repair

A 100-year design frequency is to be used for widening, rehabilitation or repair of an existing bridge, and for extension of an existing culvert. The same level of hydraulic analysis is performed as would be for a new bridge or culvert. The purpose of this analysis is to confirm the hydraulic adequacy of the structure. Variances from the design criteria given below may be required; however, if the hydraulic capacity of the structure is found to be severely deficient, consideration should be given to replacement of the structure.

Temporary bridges

Temporary bridges are designed to pass the 10-year discharge and meet backwater criteria. National Flood Insurance Program (NFIP) regulations are also to be considered in designing temporary bridges. See Section 8.3 on the NFIP for additional considerations.

Basic flood

The basic flood is a flood having a recurrence interval of 100 years. Hydraulic data for the basic flood, including discharge, high water surface elevation, and estimated backwater are included on the plans if the design frequency is other than 100-year.

Overtopping discharge

The overtopping discharge is the lowest discharge that overtops the lowest point in the roadway. The overtopping frequency is the recurrence interval of the overtopping discharge.

If the overtopping discharge is less than the 500-year discharge, the overtopping discharge and overtopping frequency shall be determined and shown on the plans. If the 500-year discharge does not overtop the roadway, the overtopping flood frequency need not be determined; however it should be noted on the plans that the overtopping flood frequency is greater than 500-years.

8.2.1.2 Backwater

Backwater is the increase in the upstream water surface level resulting from an obstruction in flow, such as a roadway fill with a bridge opening placed on the floodplain. The normal water surface elevation is the elevation of the water surface across the flood plain without the bridge, culvert, or roadway fill. Backwater is measured above the normal water surface elevation, and is the maximum difference between the normal water surface elevation and the water surface elevation resulting from the obstruction to flow as shown in Figure 8.2.1.1. The design high water surface elevation is the normal water surface elevation at the centerline of the proposed roadway for the design flood discharge.

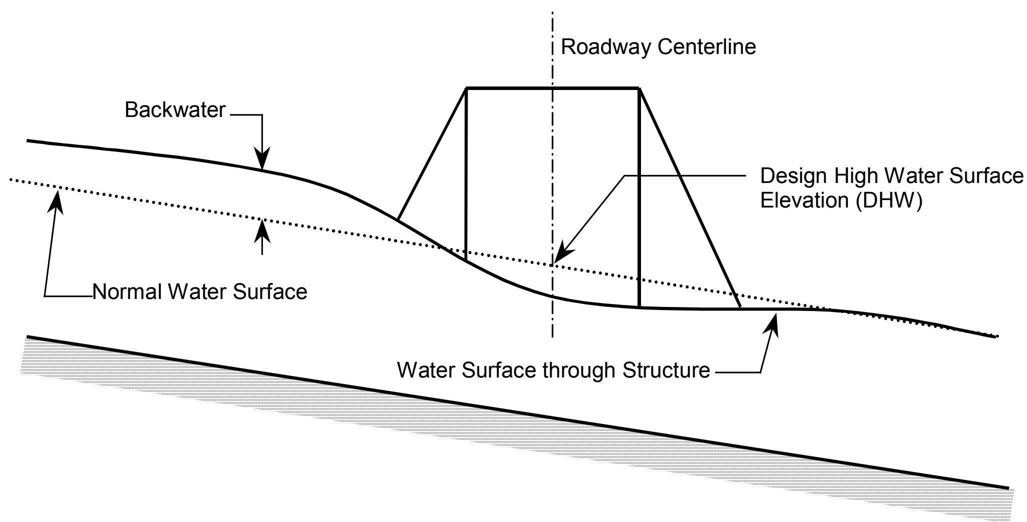

Figure 8.2.1.1 Measurement of Backwater

Allowable Backwater

The maximum allowable backwater for bridges and culverts is 1.0 ft (300 mm) at the design discharge; however, more stringent backwater criteria apply to crossings of a National Flood Insurance Program (NFIP) regulatory floodway. Construction within an NFIP regulatory floodway can cause no increase in base flood elevations (BFE's). See Section 8.2.3 for additional information on the NFIP. The maximum backwater of 1.0 ft (300 mm) applies to sites covered by the NFIP but which do not have a regulatory floodway, and to all sites not covered by the NFIP.

In addition to these backwater criteria, the designer shall check for risk of significant damage to property upstream of the crossing and insure that the structure will not significantly increase flooding of upstream properties. Where risk to upstream properties is significantly increased, consideration should be given to lowering allowable backwater to less than 1.0 ft (300 mm).

Backwater from Another Stream

The term "backwater" is also used to describe the increase in water surface elevations near the confluence of one stream with another, caused by flood conditions on the larger stream. In this case, the water surface elevation of the larger stream causes the obstruction to flow for the smaller stream and results in backwater on the smaller stream. When backwater from another stream causes water surface elevations higher than the design high water surface elevation, both elevations shall be shown on the plans.

8.2.1.3 Freeboard

Freeboard is the required clearance between the lower limit of superstructure and the design high water surface elevation. An appropriate amount of freeboard allows for the safe passage of ice and debris through the structure. The required structure grade elevation is obtained by adding freeboard and superstructure depth to the design high water elevation. Minimum freeboard is given in Table 8.2.1.1.

Table 8.2.1.1 Minimum Freeboard

Structure Type	Minimum Freeboard
Headwater:	
Bridges with Drainage Area ≥ 20 mi^2 (50 km^2)	2.0 ft (1.0 m)
Bridges with Drainage Area < 20 mi^2 (50 km^2)	1.0 ft (0.5 m)
Temporary Bridges	1.0 ft (0.5 m)
Culverts	0.0 ft (0.0 m)
Backwater from another stream:	
Bridges	1.0 ft (0.5 m)
Culverts	0.0 ft (0.0 m)

8.2.1.4 Velocity

Average velocity through the structure and average velocity in the channel shall be evaluated to insure they will not result in damage to the highway facility or an increase in damage to adjacent properties. Average velocity through the structure is determined by dividing the total discharge by the total area below design high water. Average velocity in the channel is determined by dividing the discharge in the channel by the area in the channel below design high water.

Acceptable velocities will depend on several factors, including the "natural" or "existing" velocity in the stream, existing site conditions, soil types, and past flooding history. Engineering judgment must be exercised to determine acceptable velocities through the structure.

Past practice has shown that bridges meeting backwater criteria will generally result in an average velocity through the structure of somewhere near 6 ft/s (2.0 m/s). An average velocity significantly different from 6 ft/s (2.0 m/s) may indicate a need to further refine the hydraulic design of the structure.

8.2.1.5 Hydraulic Performance Curve

The hydraulic performance of the proposed structure shall be evaluated at various discharges, including the 10-, 50-, 100-, and 500-year discharges. The risk of significant damage to adjacent properties by the resulting velocity and backwater for each of these discharges shall be evaluated.

8.2.1.6 Flow Distribution

Flow distribution refers to the relative proportions of flow on each overbank and in the channel. The existing flow distribution should be maintained whenever possible. Maintaining the existing flow distribution will eliminate problems associated with transferring flow from one side of the stream to the other, such as significant increases in velocity on one overbank. One-dimensional water surface profile models are not intended to be used in situations where the flow distribution is significantly altered through a structure. Maintaining the existing flow distribution generally results in the most hydraulically efficient structure.

8.2.1.7 Hydraulic Considerations for Bridge Layout

Abutments shall be placed such that spill fill slopes do not infringe upon the channel; the toes of the spill fill slopes may be no closer to the center of the channel than the toe of the channel banks. The Soil Survey provided by the Materials Division gives minimum spill fill slopes based on slope stability criteria. The minimum bridge length for stability criteria is thus determined by projecting the stability slopes outward from the toes of the channel slopes as shown in Figure 8.2.1.2. For structures crossing an NFIP regulatory floodway, abutments shall be placed such that the toes of the spill fill slopes are outside the floodway limits.

Piers should not be placed in the channel except where absolutely necessary. Where possible, piers are to be placed no closer to the center of channel than the toe of the channel banks. When the proposed bridge length is such that piers in the channel are necessary, the number of piers in the channel shall be kept to a minimum.

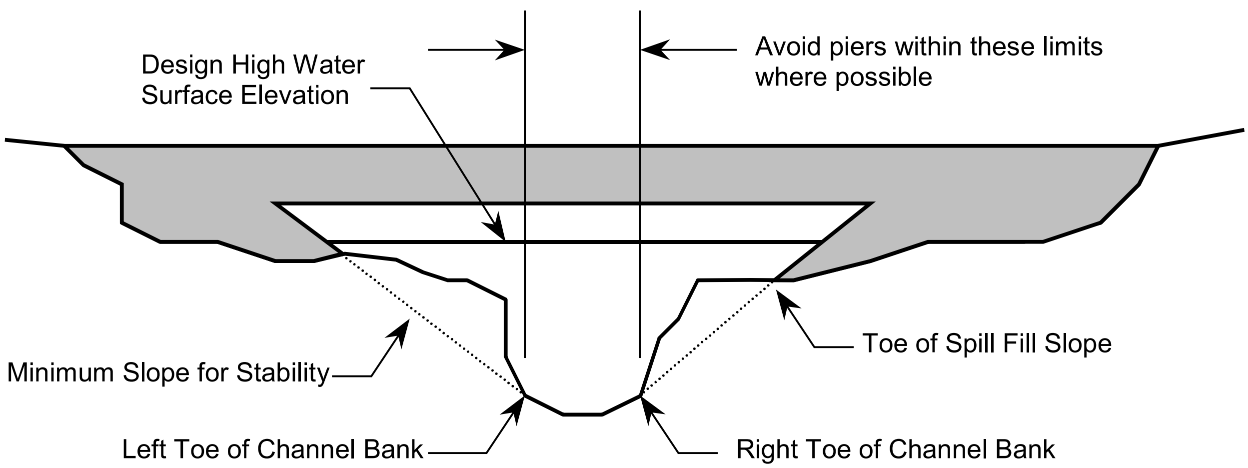

Figure 8.2.1.2. Minimum Length Bridge for Stability Criteria

Bents shall be skewed where necessary to align piers to the flow direction, at the design discharge, to minimize the disruption of flow and to minimize scour at piers. For stream crossings, skew angles less than 10 degrees are not typically used, and skew angles should be evenly divisible by 5 degrees.

When replacing an existing bridge, the bridge memorandum and design layout should note whether the existing roadway fill is to be removed. Normally, the designer should specify that the existing fill is to be removed to the natural ground line to the limits of the design high water.

8.2.1.8 Scour

Hydraulic analysis of a bridge design requires evaluation of the proposed bridge's vulnerability to potential scour. Unanticipated scour at bridge piers or abutments can result in rapid bridge collapse and extreme hazard and economic hardship.

Bridge scour is composed of several separate yet interrelated components, including long term profile changes, contraction scour and local scour. Total scour depths are obtained by adding all of these components.

Lateral channel movement must also be considered in design of bridge foundations. Stream channels typically are not fixed in location and tend to move laterally. Consideration should be given to setting foundation elevations on the overbanks at the same elevation as foundations in the channel when significant lateral channel migration is expected.

The bottom of footing elevations should be set at or below the calculated total scour depth, provided the calculated depths appear reasonable. A minimum bottom of footing elevation of 9.0 ft (3.0m) below the existing ground or channel bottom shall be used. The bottom of footing elevation shall remain the same whether a seal course is used or not; do not adjust the bottom of footing if a seal course is used. Considerable exercise of engineering judgement may be required in setting these footing depths.

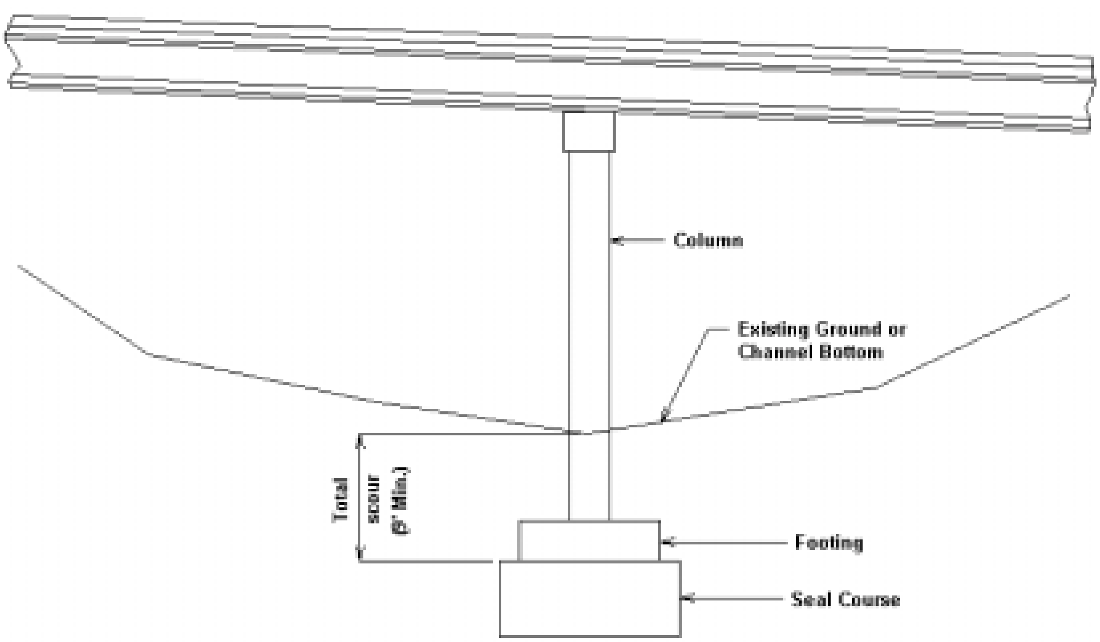

Figure 8.2.1.3. Total Scour

8.2.1.9 Bank/Channel Stability

Bank and channel stability must be considered during the design process. HEC-20 (1) provides additional information on factors affecting streambank and channel stability, and provides procedures for analysis of streambank and channel stability. At a minimum, a qualitative analysis (HEC-20 Level 1) of stream stability shall be performed. If this qualitative analysis indicates a high potential for instability at the site, a more detailed analysis may be warranted.

See the AASHTO Highway Drainage Guidelines Volume VI (2) and HEC-20 for additional information.

8.2.1.10 Coordination, Permits, and Approvals

The interests of other agencies must be considered in the evaluation of a proposed stream-crossing system, and cooperation and coordination with these agencies must be undertaken. Coordination with the State Emergency Management Agency (SEMA), the U.S. Coast Guard, the U.S. Army Corps of Engineers, and the Department of Natural Resources is required.

Required permits include:

- U.S. Coast Guard permits for construction of bridges over navigable waterways
- Section 404 permits for fills within waterways of the United States from the U.S. Army Corps of Engineers
- Section 401 Water Quality Certification permits from the Missouri Department of Natural Resources
- floodplain development permits from the State Emergency Management Agency (SEMA).

Section 404 and Section 401 permits are obtained by the Design Division. U.S. Coast Guard permits and floodplain development permits are obtained by the Bridge Division.

Copies of approved U.S. Coast Guard permits and floodplain development permit/applications are sent to the District, with a copy to the Design Division.

See Section 8.2.3 of this manual and Section 4-09 of the Project Development Manual for more information on the required permits.

8.2.1.11 Design Variance

The Division Engineer, Bridge, must approve any exception to these design criteria. A complete explanation of the basis for the design variance must be provided, including cost justification and details on how the variance will affect adjacent properties. Exceptions to these design criteria for projects on Interstate routes must also be approved by FHWA.

8.2.2 Hydraulic Design Process

8.2.2.1 Overview

The hydraulic design process begins with the collection of data necessary to determine the hydrologic and hydraulic characteristics of the site. The hydraulic design process then proceeds through the hydrologic analysis stage, which provides estimates of peak flood discharges through the structure. The hydraulic analysis provides estimates of the water surface elevations required to pass those peak flood discharges. A scour analysis provides an estimate of the required depth of bridge foundations. A risk assessment is performed for all structures, and when risks to people, risks to property, or economic impacts are deemed significant, a least total economic cost analysis shall be performed to insure the most appropriate and effective expenditure of public funds. Finally, proper documentation of the hydraulic design process is required.

The level of detail of the hydrologic and hydraulic analyses shall remain consistent with the site importance and with the risk posed to the highway facility and adjacent properties by flooding.

8.2.2.2 Data Collection

The first step in hydraulic design is collecting all available data pertinent to the structure under consideration. Valuable sources of data include the bridge survey; aerial photography and various maps; site inspections; soil surveys; plans, surveys, and computations for existing structures; and flood insurance study data.

Bridge survey

The bridge survey is prepared by district personnel and provides information regarding existing structures, nearby structures on the same stream, and streambed and valley characteristics including valley cross-sections along the centerline of the proposed structure, valley cross-sections upstream and downstream of the proposed structure, and a streambed profile through the proposed structure.

Location of the surveyed valley cross-sections is an important factor in developing the best possible water surface profile model for the proposed structure. For this reason, inclusion of the bridge survey as an agenda item on an initial core team meeting to discuss appropriate location of the valley cross-sections is recommended.

Photographs and maps

Aerial photography, USGS topographic maps, and county maps should be consulted to determine the geographic layout of the site. Aerial photographs, in particular, can provide information on adjacent properties that may be subjected to increased risk of flood damage by the proposed structure, and may be available from the MoDOT photogrammetric section.

Site Inspection

A site inspection is a vital component of the hydrologic and hydraulic analyses, and is especially important for those sites subjected to risk of significant flood damage. A visit to the proposed site will provide the following information:

- selection of roughness coefficients
- evaluation of overall flow directions
- observation of land use and related flood hazards
- geomorphic observations (bank and channel stability)
- high-water marks
- evidence of drift and debris
- interviews with local residents or construction and maintenance engineers on flood history

Photographs taken during the site visit provide documentation of existing conditions and will aid in later determination of hydraulic characteristics.

Flood Insurance Study data

If a Flood Insurance Study (FIS) has been performed for the community in which the structure is proposed, the FIS may provide an additional data source. The FIS may contain information on peak flood discharges, water surface profile elevations, and information on regulatory floodways.

Data review

After all available data have been compiled, the data should be reviewed for accuracy and reliability. Special attention should be given to explaining or eliminating incomplete, inconsistent or anomalous data.

8.2.2.3 Hydrologic Analysis

Peak flood discharges are determined by one of the following methods. If the necessary data is available, discharges should be determined by all methods and engineering judgment used to determine the most appropriate.

Historical USGS stream gage data.

Numerous USGS recording stream gages have been maintained for many years on selected Missouri streams. For proposed structures at or near one of these gages, the gage data can be used in estimating discharge. When sufficient years of data have been collected at a stream gage, the data may be statistically analyzed to estimate discharge for the selected design flood frequency.

Stream gage data is available on the Internet at

http://wwwdmorll.er.usgs.gov/

under *Historical Streamflow Data*.

Gage data is analyzed by Log-Pearson Type III regression analysis to determine the discharges associated with the relevant return periods. See Water Resources Council Bulletin #17B (3) for details on this analysis method. A computer program for the analysis is available.

One statistical parameter computed in the Log Pearson analysis is the skew coefficient of the distribution of the stream gage data. Skew coefficients for the data from stream gages in Missouri are typically between -0.1 and -0.4 when sufficient years of record are available. Skew coefficients outside this range may indicate an insufficient length of record or an analysis affected by outliers in the data. In this case, other methods of determining discharges will likely provide better estimates.

Stream gage data from gages at some distance from the site on the same watershed and stream gage data from nearby hydrologically similar watersheds may also be used to estimate discharges. Discharges obtained from this type of data should be compared with discharges obtained by other methods and not given the same weight as discharges obtained from data from a stream gage at the proposed site. Better estimates of discharge using this method may be obtained by repeating the procedure for several nearby gages and averaging the results. This method should not be used when drainage areas differ by more than 50% or at sites more than 50 miles (80 km) from the stream gage(s).

Transposition of discharges from one basin to another, or from one location to another within the same watershed, is accomplished using the following equation:

$$Q_1 = Q_2 \cdot \left(\frac{A_1}{A_2} \right)^k$$

where:

Q_1 = discharge for drainage basin 1 (cfs or m^3/s)
A_1 = drainage area for drainage basin 1 (mi^2 or km^2)
Q_2 = discharge for drainage basin 2 (cfs or m^3/s)
A_2 = drainage area for drainage basin 2 (mi^2 or km^2)
k = exponent = 0.5 to 0.7

NFIP Flood Insurance Study discharges

NFIP Flood Insurance Studies typically include estimates of 10-, 50-, 100- and 500-year discharges for streams studied by detailed methods. These discharges may be more accurate than those obtained by other methods if the FIS discharges were determined through a detailed hydrologic study, such as an HEC-1 or TR-20 hydrologic model. In some instances, the FIS discharges may have been determined using an older version of the USGS regression equations. These discharges should not be used. Careful review of the FIS report will disclose the level of detail used in the hydrologic study.

USGS Rural Regression equations

These equations were developed in 1995 by the United States Geological Survey in Rolla (4). Data from 278 gaged sites in Missouri were analyzed to determine flood magnitudes with recurrence intervals of 2, 5, 10, 25, 50, 100 and 500 years. The resulting magnitudes were then related to hydrologic region, drainage area and average main-channel slope by a statistical analysis to provide the regression equations.

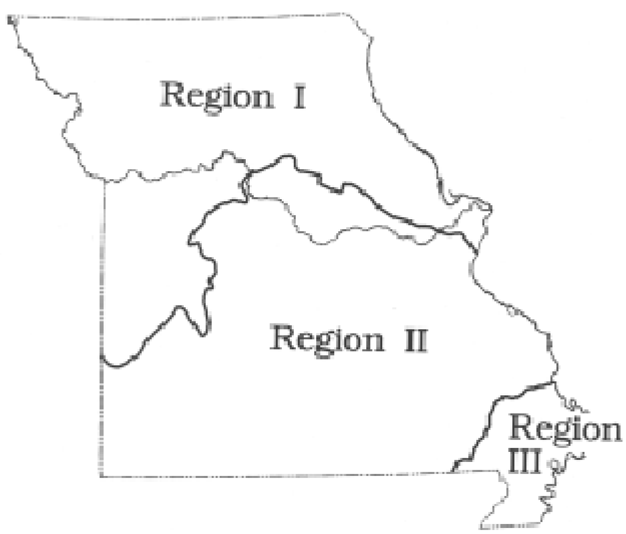

Figure 8.2.2.1 Missouri's Hydrologic Regions

The state is divided into three hydrologic regions, each with its own set of regression coefficients. The three regions are shown in Figure 8.2.2.1 and are described as follows:

Region I - Central Lowlands - "Characterized by meandering stream channels in wide and flat valleys resulting in long and narrow drainage patterns with local relief generally between 20 to 150 ft (15 to 45 m). Elevations range from about 600 ft (180 m) above sea level near the Mississippi River to about 1200 ft (370 m) above sea level in the northwest parts of the region"

Region II - Ozark Plateaus - "Characterized by streams that have cut narrow valleys 200 to 500 ft (60 to 150 m) deep, resulting in sharp rugged ridges that separate streams, with local relief generally ranging from 100 to 500 ft (30 to 150 m). The drainage patterns are described as dendritic (tree shaped) with main-channel gradients steeper than elsewhere in Missouri, and karst features are locally prominent in much of the region. Elevations (generally) range from 800 to 1700 ft (240 to 520 m) above sea level"

Region III - Mississippi Alluvial Plain - "A relatively flat area of excellent farmland. Virtually all the area is drained by a series of man-made drainage ditches that slope southward at an average of about 1.5 ft/mile (0.28 m/km). Elevations range from 200 to 300 ft (60 to 90 m) above sea level with local relief seldom exceeding 30 ft (10 m)".

For ungaged natural floodflow sites, flood magnitudes having recurrence intervals of 2, 5, 10, 25, 50, 100 and 500 years are computed using appropriate values of the contributing drainage basin area and slope in the following equation:

$$Q_f = a \cdot A^{b_1} S^{b_2}$$

where:

Q_f = Flood magnitude for flood frequency f (cfs or m^3/s)
a = Regression constant
b_1, b_2 = Regression coefficients
A = Basin drainage area (mi^2 or km^2)
S = Valley slope (ft/mi or m/km)

The values of a, b_1, and b_2 are given in Table 8.2.2.1 below. A computer program is available to assist in performing these calculations.

Table 8.2.2.1 Coefficients for USGS Rural Regression Equations

Flood Frequency,	English			Metric		
f	*a*	b_1	b_2	*a*	b_1	b_2
Region I						
2	69.4	0.703	0.373	1.87	0.703	0.373
5	123	0.690	0.383	3.42	0.690	0.383
10	170	0.680	0.378	4.73	0.680	0.378
25	243	0.668	0.366	6.70	0.668	0.366
50	305	0.660	0.356	8.34	0.660	0.356
100	376	0.652	0.346	10.2	0.652	0.346
500	569	0.636	0.321	15.0	0.636	0.321
Region II						
2	77.9	0.733	0.265	1.71	0.733	0.265
5	99.6	0.763	0.355	2.46	0.763	0.355
10	117	0.774	0.395	3.06	0.774	0.395
25	140	0.784	0.432	3.86	0.784	0.432
50	155	0.789	0.453	4.40	0.789	0.453
100	170	0.794	0.471	4.95	0.794	0.471
500	203	0.804	0.503	6.18	0.804	0.503
Region III						
2	88.0	0.658	n/a	1.33	0.658	n/a
5	145	0.627	n/a	2.26	0.627	n/a
10	187	0.612	n/a	2.96	0.612	n/a
25	244	0.595	n/a	3.92	0.595	n/a
50	288	0.585	n/a	4.67	0.585	n/a
100	334	0.576	n/a	5.47	0.576	n/a
500	448	0.557	n/a	7.47	0.557	n/a

Drainage Area

Drainage area (A) in mi^2 (km^2), can be obtained by determining the area contributing surface flows to the site as outlined along the drainage divide on the best available topographic maps.

Valley slope

Valley slope (S) in feet per mile (meters per kilometer) is the average slope between points 10 percent and 85 percent of the distance along the main-stream channel from the site to the drainage divide. Distance is measured by setting draftsman's dividers at 0.1 mile (0.1 km) spread and stepping along the main channel. The main channel is defined above stream junctions as the one draining the largest area. The elevation difference between the 10- and 85-percent points is divided by the distance between the points to evaluate the slope.

Limitations of equations

The USGS Rural Regression Equations may be used to estimate magnitude and frequency of floods on most Missouri streams provided the drainage area and slope are within the limits given in Table 8.2.2.2.

However, the equations are not applicable for basins where manmade changes have appreciably changed the flow regimen, the main stems of the Mississippi and Missouri Rivers, and areas near the mouth of streams draining into larger rivers where a backwater effect is experienced.

Table 8.2.2.2 Limitations of USGS Rural Regression Equations

Region	Area limits (mi^2)	Slope limits (ft/mi)
I	0.13 to 11,500	1.35 to 150
II	0.13 to 14,000	1.2 to 279
III	0.48 to 1040	n/a

Region	Area limits (km^2)	Slope limits (m/km)
I	0.34 to 29800	0.26 to 28
II	0.34 to 36000	0.23 to 53
III	1.24 to 2690	n/a

USGS Urban Regression Equations

The USGS Rural Regression Equations given above are not applicable to urban watersheds where manmade changes have appreciably changed the flow regimen. A set of USGS Urban Regression Equations were developed in 1986 by the United States Geological Survey in Rolla for use in urban watersheds (5). Data from 37 gaged sites in both urban and rural locations in Missouri were analyzed to determine flood magnitudes with recurrence intervals of 2, 5, 10, 25, 50, and 100 years. The resulting magnitudes were then related to drainage area and average main-channel slope to provide the regression equations.

An urban watershed may be defined as a drainage basin in which manmade developments in the form of impervious surfaces and/or storm drainage systems have substantially altered the basin's natural response to rainfall. Urbanization of a natural watershed progresses in one of two ways. First, the addition of impervious surfaces in the form of roads, streets, parking lots and roofs will prevent infiltration of rainfall into the covered soil surface, thus increasing the total volume and peak rate of runoff from a given rainfall volume. Second, to protect the now valuable property in a developed watershed from this increased peak and volume of runoff, storm drainage systems are installed. The installation of a storm drainage system does not increase the volume of runoff, but modifies the time distribution of runoff. Thus, when storm water drainage systems are installed, the time of concentration of the watershed is decreased. Therefore, storm water drainage systems have the effect of removing a given volume of runoff in a shorter period of time, thus increasing the peak rate of runoff.

All hydraulic design in urban areas should consider the effect of increasing development throughout the projected life of the structure. Information on planned future development may be available from local agencies.

Peak discharges can be estimated at urban locations using either of the two equations presented below. Both equations give peak discharge as a function of drainage area and a characteristic of urbanization: either basin development factor (*BDF*) or percentage of impervious area. Choice of which equation to use should depend on whether it is easier to determine *BDF* or percentage of impervious area for a given basin. Either of the equations should provide comparable results.

The equation utilizing the basin development factor is given as:

$$Q_f = a \cdot A^{b_1} \cdot (13 - BDF)^{b_2}$$

where:

Q_f = Flood magnitude for flood frequency f (cfs or m^3/s)
a = Regression constant
b_1, b_2 = Regression coefficients
A = Basin drainage area (mi^2 or km^2)
BDF = Basin development factor

The values of a, b_1, and b_2 may be obtained from Table 8.2.2.3.

Table 8.2.2.3 Coefficients for USGS Urban BDF Regression Equations

Flood Frequency,	English			Metric		
f	a	b_1	b_2	a	b_1	b_2
2	801	0.747	-0.400	11.1	0.747	-0.400
5	1150	0.746	-0.318	16.0	0.746	-0.318
10	1440	0.755	-0.300	19.9	0.755	-0.300
25	1920	0.764	-0.307	26.3	0.764	-0.307
50	2350	0.773	-0.319	31.9	0.773	-0.319
100	2820	0.783	-0.330	37.9	0.783	-0.330
500	n/a	n/a	n/a	n/a	n/a	n/a

The equation utilizing Percentage of Impervious Area is given as:

$$Q_f = a \cdot A^{b_1} \cdot I^{b_2}$$

where:

Q_f = Flood magnitude for flood frequency f (cfs or m^3/s)
a = Regression constant
b_1, b_2 = Regression coefficients
A = Basin drainage area (mi^2 or km^2)
I = Percentage of Impervious Area

The values of a, b_1, and b_2 may be obtained from Table 8.2.2.4.

Table 8.2.2.4 Coefficients for USGS Urban % Impervious Area Regression Equations

Flood Frequency, f	English			Metric		
	a	b_1	b_2	A	b_1	b_2
2	224	0.793	0.175	2.98	0.793	0.175
5	424	0.784	0.131	5.69	0.784	0.131
10	560	0.791	0.124	7.47	0.791	0.124
25	729	0.800	0.131	9.64	0.800	0.131
50	855	0.810	0.137	11.2	0.810	0.137
100	986	0.821	0.144	12.8	0.821	0.144
500	n/a	n/a	n/a	n/a	n/a	n/a

A computer program is available to assist in performing these calculations.

Basin Development Factor

The basin development factor (*BDF*) is determined by dividing the drainage basin into thirds (subareas). Each subarea of the basin is then evaluated for four aspects of urbanization. For each of the four criteria, a value of either 1 (if the subarea meets the criteria) or 0 (if the subarea does not meet the criteria) is assigned. The *BDF* is the sum of the values for each of the four criteria and for each third of the basin. A maximum *BDF* of twelve results when each of the three subareas meets each of the four criteria for urbanization described below:

• Channel Improvements - channel improvements such as straightening, enlarging, deepening, and clearing have been made to at least 50 percent of the main channel and principal tributaries.

• Channel Linings - more than 50 percent of the main channel and principal tributaries has been lined with an impervious material. (Note that the presence of the channel linings also implies the presence of channel improvements.)

• Storm Drains or Storm Sewers - more than 50 percent of the secondary tributaries of a subarea consists of storm drains or storm sewers.

• Curb-and Gutter Streets - more than 50 percent of a subarea is urbanized and more than 50 percent of the streets and highways in the subarea are constructed with curbs and gutters.

The valid range for *BDF* is 0 to 12. Typical drainage-basin shapes and the method of subdivision into thirds are shown in Figure 8.2.2.2.

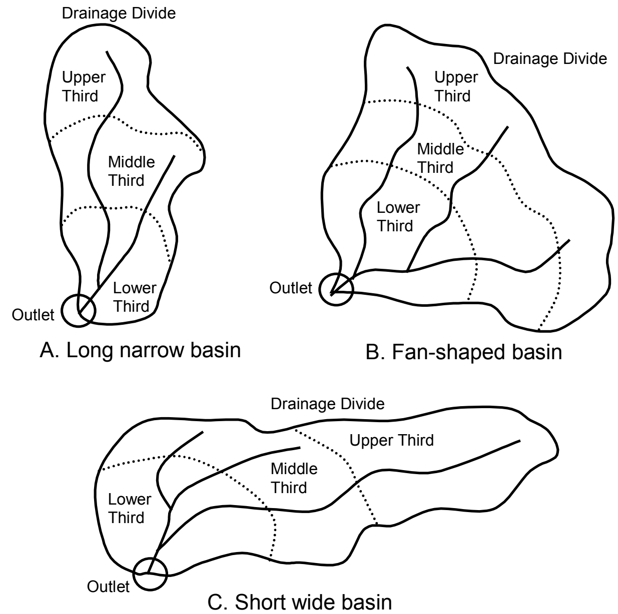

Figure 8.2.2.2. Typical drainage basin shapes and subdivision of basins into thirds. (Becker, 1986)

Percentage of Impervious Area

The percentage of impervious area (I) is the portion of the drainage area into which water cannot infiltrate because of buildings, parking lots, streets and roads, and other impervious areas within an urban basin. The variable, I, is determined from the best available maps or aerial photos showing impervious surfaces. Field inspection to supplement the maps may be useful.

If the percentage of impervious area cannot be determined directly, a reasonable estimate may be obtained using 7-1/2 minute topographic maps and a relationship between developed area and impervious area. The drainage divide is outlined on the map, then the drainage area is divided into two subareas, open area and developed (urban) area. Open area consists of all undeveloped land, which may include scattered farmhouses and buildings, scattered single-family housing and paved roads without significant development along the road. Developed areas include single- or multi-family housing structures, large business and office buildings, shopping centers, extensively industrialized areas, and schools. When delineating developed areas, it is important to include those areas devoted to paved parking lots around buildings. Once the amount of developed area has been determined, it can be converted into a percentage developed area (PDA) by dividing by the basin drainage area and multiplying by 100. The percentage of impervious area can then be obtained using the following equation:

$$I = 2.03 \cdot PDA^{0.618}$$

The valid range for I is 1.0 percent to 40 percent. The values for both I and PDA are entered as percents (i.e. I = 29 for 29% impervious area and PDA = 75 for 75% developed area.)

Limitations of Equations

The USGS Urban Regression Equations may be used to estimate magnitude and frequency of floods on most urban Missouri streams, for drainage areas between 0.25 and 40 mi^2 (0.65 and 100 km^2) with valley slopes between 8.7 to 120 ft/mi (1.7 and 22 m/km), provided that the flood flows are relatively unaffected by manmade works such as dams or diversions.

Other methods

Other methods of determining peak flood discharges include the Corps of Engineers' HEC-1 and HEC-HMS hydrologic modeling software programs, the SCS TR-20 hydrologic modeling software program, and the SCS TR-55 Urban Hydrology for Small Watersheds method.

Use of these alternate methods should be limited to situations where the methods given above are deemed inappropriate or inadequate.

8.2.2.4 Hydraulic Analysis of Bridges

The Corps of Engineers Hydrologic Engineering Center's River Analysis System (HEC-RAS) shall be used to develop water surface profile models for the hydraulic analysis of bridges. Documentation on the use of HEC-RAS is available in references (6), (7), and (8).

Hydraulic design of bridges requires analysis of both the "natural conditions" and the "proposed conditions" at the site. In order to show that structures crossing a NFIP regulatory floodway cause no increase in water surface elevations, it is also necessary to analyze the "existing conditions."

For these reasons, water surface profile models for bridges shall be developed for three conditions:

- Natural conditions - Includes natural channel and floodplain, including all modifications made by others, but without MoDOT structures
- Existing conditions - Includes natural conditions and existing MoDOT structure(s)
- Proposed conditions - Includes natural conditions, existing MoDOT structures if they are to remain in place, and proposed MoDOT structure(s)

Backwater is determined by comparing the water surface elevations upstream of the structure for either existing conditions or proposed conditions to the corresponding water surface elevation for the natural conditions.

For bridges near a confluence with a larger stream downstream of the site, additional models may be required. The water surface profile and resulting backwater should be evaluated both with and without backwater from the larger stream. The higher backwater resulting from the proposed structure shall be considered to control.

The hydraulic model in HEC-RAS is based on an assumption of one-dimensional flow. If site conditions impose highly two-dimensional flow characteristics (i.e. a major bend in the stream just upstream or downstream of the bridge, very wide floodplains constricted through a small bridge opening, etc.), the adequacy of these models should be considered. A two-dimensional model may be necessary in extreme situations.

Design high water surface elevation

The design high water surface elevation is the normal water surface elevation at the centerline of the roadway for the design flood discharge. This elevation may be obtained using the slope-area method or from a "natural conditions" water surface profile.

Slope-area method - The slope-area method applies Manning's equation to a natural valley cross-section to determine stage for a given discharge. Manning's equation is given as:

$$Q = \frac{1.486}{n} \cdot A \cdot R^{2/3} \cdot S_o^{\ 1/2}$$

where:

Q	= Discharge (cfs or m^3/s)
n	= Manning's roughness coefficient
A	= Cross-sectional area (ft^2 or m^2)
R	= Hydraulic radius = A/P
P	= Wetted perimeter (ft or m)
S_o	= Hydraulic gradient (ft/ft or m/m)

In order to apply Manning's equation to a natural cross-section, the cross-section must be divided into sub-sections. The cross-section should be divided at abrupt changes in geometry and at changes in roughness characteristics.

For a given water surface elevation, the discharge can be determined directly from Manning's equation. Determination of the water surface elevation for a given discharge requires an iterative procedure.

The slope-area method should not be used with the roadway centerline valley cross-section to determine the design high water surface elevation when the centerline cross-section is not representative of the stream reach, such as when the new alignment follows or is very near the existing alignment. The centerline cross-section should also not be used when the centerline cross-section is not taken perpendicular to the direction of flow, such as when the alignment is skewed to the direction of flow or is on a horizontal curve. In these cases, an upstream or downstream valley cross-section should be used to determine the design high water surface elevation. The water surface elevation for an upstream or downstream valley cross-section can be translated to the roadway centerline by subtracting or adding, respectively, the hydraulic gradient multiplied by the distance along the stream channel from the valley cross-section to the roadway centerline.

A computer program is available to assist in making the slope-area calculations.

Roughness Coefficients

Roughness coefficients (Manning's "n") are selected by careful observation of the stream and floodplain characteristics. Proper selection of roughness coefficients is very significant to the accuracy of computed water surface profiles. The roughness coefficient depends on a number of factors including surface roughness, vegetation, channel irregularity, and depth of flow. It should be noted that the discharge in Manning's equation is inversely proportional to the roughness coefficient; a 10% decrease in roughness coefficient will result in a 10% increase in the discharge for a given water surface elevation.

It is extremely important that roughness coefficients in overbank areas be carefully selected to represent the effective flow in those areas. There is a general tendency to overestimate the amount of flow occurring in overbank areas, particularly in broad, flat floodplains. Increasing the roughness coefficients on overbanks will increase the proportion of flow in the channel, with a corresponding decrease in the proportion of flow on the overbanks.

References (9), (10), and (11) provide guidance on the selection of roughness coefficients.

Hydraulic gradient (streambed slope):

The hydraulic gradient, S_o, is the slope of the water surface in the vicinity of the structure. It is generally assumed equal to the slope of the streambed in the vicinity of the structure. Note that the hydraulic gradient is typically much smaller than the valley slope used in the USGS regression equations. Hydraulic gradient is a localized slope, while valley slope is the average slope of the entire drainage basin.

Hydraulic gradient is determined by one of two methods, depending on drainage area:

- For drainage areas less than 10 mi^2 (25 km^2), the gradient is determined by fitting a slope to the streambed profile given on the bridge survey.

- For drainage areas greater than 10 mi^2 (25 km^2), the gradient is determined from USGS 7.5 minute topographic maps by measuring the distance along the stream between the nearest upstream and downstream contour crossings of the stream. The hydraulic gradient is then given by the vertical distance between contours divided by the distance along the stream between contours. Dividers set to 0.1 mi or 0.1 km should be used to measure the distance along the stream.

Overtopping discharge and frequency

The overtopping flood frequency of the stream crossing system - roadway and bridge - shall be determined if the overtopping frequency is less than 500-years. An approximate method of determining the overtopping discharge uses the slope-area method given above and setting the stage to the elevation of the lowest point in the roadway. A more accurate method involves using a trial-and-error procedure, adjusting the discharge in the HEC-RAS proposed conditions model until flow just begins to overtop the roadway. The overtopping frequency can then be estimated by linear interpolation from previously developed discharge-frequency data.

Waterway Enlargement

There are situations where roadway and structural constraints dictate the vertical positioning of a bridge and result in small vertical clearances between the low chord and the ground. In these cases, significant increases in span length provide small increases in effective waterway opening. It is possible to improve the effective waterway area by excavating a flood channel through the reach affecting the hydraulic performance of the bridge. This is accomplished by excavating material from the overbanks as shown in Figure 8.2.2.3; enlargement of the channel itself is avoided where possible as excavation below ordinary high water is subject to 404 permit requirements.

A similar action may be taken to compensate for increases in water surface elevations caused by bridge piers in a floodway.

There are, however, several factors that must be accommodated when this action is taken.

- The flow line of the flood channel must be set above the ordinary high water elevation.
- The flood channel must extend far enough upstream and downstream of the bridge to establish the desired flow regime through the affected reach.
- Stabilization of the flood channel to prevent erosion and scour should be considered.

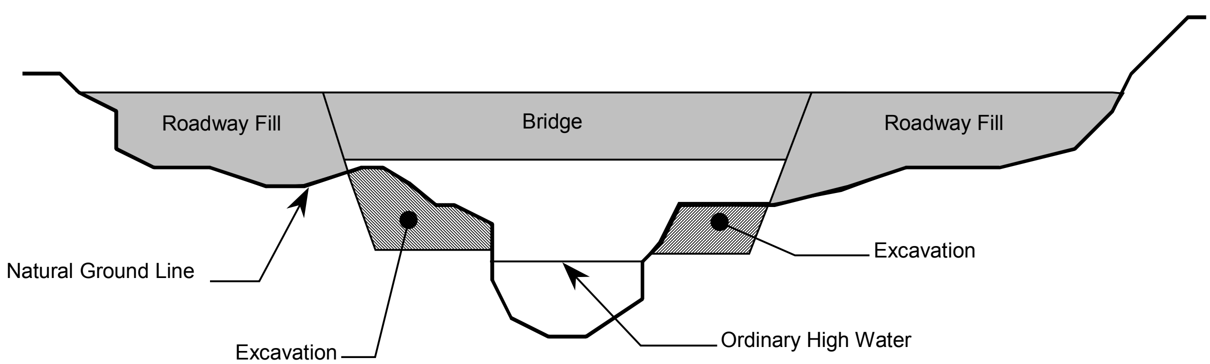

Figure 8.2.2.3 Typical Excavation for a Flood Channel

8.2.2.5 Hydraulic Analysis of Culverts

The FHWA HY-8 computer program shall be used for the analysis and design of culverts. The hydraulic design of highway culverts is based on the theory and procedures presented in *Hydraulic Design of Highway Culverts - HDS No. 5* (12).

Flow Type

A culvert barrel may flow full over its entire length or may flow partly full. Full flow results in pressure flow within the culvert barrel, while partly full flow is a form of free surface or open channel flow.

Flow Control

Culverts may operate under one of two types of flow control; either inlet control or outlet control. Type of control is a function of the location of the control section.

Inlet control exists when the headwater elevation depends on the culvert entrance configuration and the barrel is capable of conveying more flow than the inlet will accept. The control section is at or just inside the culvert inlet. The water surface passes through critical depth at this location, and flow is supercritical in the culvert barrel immediately downstream.

Outlet control exists when the headwater elevation depends on the culvert barrel geometry or downstream conditions and the inlet is capable of conveying more flow than the barrel will accept. The control section is either at the downstream end of the barrel or further downstream.

Culvert design requires calculation of the headwater required for both inlet control and outlet control conditions. The higher upstream headwater "controls" and determines the type of flow in the culvert for a given discharge and tailwater condition. Changing either discharge or tailwater depth may cause a culvert to switch from one type of flow control to the other.

Headwater vs Backwater

Headwater is defined as the depth of the upstream water surface measured from the invert at the culvert entrance. The invert is the lowest point inside the culvert at a particular cross-section.

Culvert backwater is the difference between the headwater elevation and the normal water surface elevation at the culvert entrance. The normal water surface elevation is generally determined at the roadway centerline, and must be projected upstream to the location at which the headwater was measured. For this reason, the design headwater elevation is dependent upon the culvert length. An iterative procedure may be necessary to determine the optimum culvert design.

HY-8 Required Input Data

Below are the data required for HY-8 culvert calculations.

Discharges - Both the design discharge and a maximum discharge are required inputs. A minimum discharge is also an optional input, but will default to zero (0). HY-8 will compute a hydraulic performance curve for a range of discharges based on the minimum and maximum discharges, and will also perform calculations for the design discharge.

Culvert Invert Data - The station and elevation for both the inlet and outlet inverts are used to determine the length of the culvert. Another option allows the input of information defining the embankment and streambed slopes. HY-8 will then use this information and the culvert height to compute the culvert length.

Roadway Data - Various information regarding the roadway is required. This information is used in weir flow calculations when the roadway is overtopped. A fixed roadway crest elevation and length can be input or coordinates describing the top of roadway can be used. The type of roadway (paved or graveled) or a weir coefficient must be input, along with the roadway width.

Tailwater Data - Tailwater depths must be provided for the various discharges used in the analysis. Several input options are available, including providing a tailwater rating curve (depth vs. discharge at the culvert outlet), providing either a natural or a prismatic channel cross-section, or providing a constant tailwater elevation. HY-8 will calculate a tailwater rating curve when a channel cross-section is input. It is recommended that either a channel cross-section be used or a tailwater rating curve be developed independently of HY-8; using a constant tailwater elevation can result in incorrect headwater elevations for the outlet control computations.

Culvert Geometry - The culvert geometry must be provided, including the number of barrels, culvert span and rise, and inlet configuration. The inlet configuration options include conventional and improved inlets, with various combinations of headwall bevels and wingwall flares for each. The "Square Edge (0 deg. flare)" and "Square Edge (30-75 deg. flare)" wingwall options should be used for standard MoDOT culverts with straight and flared wingwalls, respectively.

Minimize culvert span

HY-8 provides an option for determining the minimum culvert span for a given culvert rise and design headwater elevation. The design headwater elevation required for this input is the normal water surface at the culvert inlet plus any allowable backwater.

Improved Inlets

For culverts operating under inlet control, cost savings may be realized by using an improved inlet. This is especially true for extremely long culverts. Side-tapered inlets are the most often used type of improved inlet. Refer to HDS No. 5 (12) for details on design of improved inlets.

8.2.2.6 Scour Analysis

Current methods of analyzing scour depths are based mainly on laboratory experiment rather than on practical field data. The results should be carefully reviewed and engineering judgment used to determine their applicability to actual field conditions.

HEC-RAS includes the ability to calculate scour depths. The methods used to calculate those depths are based on the FHWA HEC-18 publication (13), and are presented below for convenience.

Long term profile changes - aggradation and degradation

Long term profile changes result from aggradation or degradation in the stream reach over time. Aggradation involves the deposition of sediment eroded from the channel and banks upstream of the site. Degradation involves the lowering or scouring of a streambed as material is removed from the streambed and is due to a deficit in sediment supply upstream. Aggradation and degradation are generally the result of changes in the energy gradient of the stream.

Aggradation and degradation over the life of a structure are difficult to predict. These long term profile changes are typically the result of human activities within the watershed including dams and reservoirs, changes in land use, gravel mining and other operations. HEC-18 (13) and HEC-20 (2) provide more information on predicting long term profile changes. Comparison of channel bottom elevations shown on plans for existing bridges to current survey data may be informative.

Contraction scour

Contraction scour is generally caused by a reduction in flow area, such as encroachment on the floodplain by highway approaches at a bridge. Increased velocities and increased shear stress in the contracted reach result in transport of bed material. Contraction scour typically occurs during the rising stage of a flood event; as the flood recedes, bed material may be deposited back into the scour hole, leaving no evidence of the ultimate scour depth.

Contraction scour may be one of two types: live-bed contraction scour or clear water contraction scour. Live-bed scour occurs when the stream is transporting bed material into the contracted section from the reach just upstream of the contraction. Clear-water scour occurs when the stream is not transporting bed material into the contracted section. The type of contraction scour is determined by comparing the average velocity of flow in the channel or overbank area upstream of the bridge opening to the critical velocity for beginning of motion of bed material, V_c. The critical velocity can be determined using the following equation:

$$V_c = C \cdot D_{50}^{1/3} \cdot y_1^{1/6}$$

where:

C = Constant =10.95 (6.19 for metric units)
V_c = Critical velocity above which bed material will be transported, ft/s (m/s)
D_{50} = Median diameter of bed material, ft (m)
Y_1 = Depth of flow in upstream channel or overbank, ft (m)

Calculated contraction scour depths greater than 6.0 ft (2.0 m) should be viewed with some skepticism. Existing field data show that contraction scour depths greater than 6.0 ft (2.0 m) are rarely encountered.

Live-bed contraction scour - Live-bed contraction scour depths can be determined using the following equations:

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \cdot \left(\frac{W_1}{W_2}\right)^{k_1}$$

$$y_s = y_2 - y_0$$

where:

y_s = Scour depth, ft (m)
y_1 = Average depth in upstream main channel, ft (m)
y_2 = Average depth in contracted section after scour, ft (m)
y_0 = Existing depth in contracted section before scour, ft (m)
Q_1 = Flow in upstream channel transporting sediment, ft^3/s (m^3/s)
Q_2 = Flow in contracted channel, ft^3/s (m^3/s)
W_1 = Bottom width of upstream main channel, ft (m)
W_2 = Bottom width of main channel in contracted section, ft (m)
n_1 = Manning's roughness coefficient for upstream main channel
n_2 = Manning's roughness coefficient for contracted channel
k_1 = Exponent depending on mode of bed material transport

The value of *k1* can be obtained from Table 8.2.2.5.

Table 8.2.2.5 Bed material transport coefficient

V_*/ω	K_1	Mode of bed material transport
< 0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
> 2.0	0.69	Mostly suspended bed material discharge

$$V_* = (g \cdot y_1 \cdot S_1)^{1/2}$$

where:

V_* = shear velocity in upstream section, ft/s (m/s)
ω = Median fall velocity of bed material based on D_{50}, ft/s (m/s)
g = Acceleration due to gravity = 32.2 ft/s^2 (9.81 m/s^2)
S_1 = Slope of energy grade line of main channel, ft/ft (m/m)
D_{50} = Median diameter of bed material, ft (m)

The fall velocity of bed material, ω, can be obtained from Figure 3 in HEC-18 (13).

The upstream cross-section is typically located either one bridge opening length or the average length of constriction upstream of the bridge. This is consistent with the required location of the approach cross-section in both HEC-RAS and WSPRO.

Clear water contraction scour - Clear-water contraction scour depths can be determined using the following equations:

$$y_2 = \left[\frac{Q_2^2}{C \cdot D_m^{2/3} \cdot W_2^2} \right]^{\frac{3}{7}}$$

$$y_s = y_2 - y_0$$

where:
C = constant = 120 (40 for metric units)
y_s = Scour depth, ft (m)
y_2 = Average depth in contracted section after scour, ft (m)
y_0 = Existing depth in contracted section before scour, ft (m)
Q_2 = Flow in contracted channel, ft^3/s (m^3/s)
W_2 = Bottom width of main channel in contracted section, ft (m)
D_m = Diameter of smallest nontransportable particle in bed material in the contracted section, assumed equal to 1.25 D_{50}, ft (m)
D_{50} = Median diameter of bed material, ft (m)

Local scour

Local scour involves removal of material from around piers, abutments and embankments and is caused by increased velocities and vortices induced by the obstruction to flow. As with contraction scour, bed material may be deposited back into the scour holes as floodwaters recede.

Pier scour and abutment scour are considered two distinct types of local scour.

Pier scour - Pier scour depths can be determined using the following equation developed at CSU:

$$y_s = 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot K_4 \cdot a^{0.65} \cdot y_1^{0.35} \cdot Fr_1^{0.43}$$

where:

y_2	= Scour depth, ft (m)
y_1	= Flow depth directly upstream of pier, ft (m)
K_1	= Correction factor for pier nose shape
K_2	= Correction factor for angle of attack of flow
K_3	= Correction factor for bed condition
K_4	= Correction factor for armoring of bed material
a	= Pier width, ft (m)
Fr_1	= Froude number directly upstream of pier = $= V_1/(g \cdot y_1)^{1/2}$
V_1	= Mean velocity of flow directly upstream of pier, ft/s (m/s)
g	= Acceleration due to gravity = 32.2 ft/s^2 (9.81 m/s^2)

The pier scour depth is limited to a maximum of 2.4 times the pier width for Froude numbers less than or equal to 0.8, and a maximum of 3.0 times the pier width for Froude numbers greater than 0.8.

For angles of attack less than 5 degrees, the value of K_1 can be obtained from Table 8.2.2.6. For angles greater than 5 degrees, the correction factor for angle of attack dominates and K_1 should be set to 1.0.

Table 8.2.2.6 Correction factor for pier nose shape

Shape of Pier Nose	K_1
Square nose	1.1
Round nose	1
Circular cylinder	1
Group of cylinders	1
Sharp nose	0.9

The correction factor for angle of attack of the flow, K_2, can be determined using the following equation:

$$K_2 = (\cos\theta + L/a \cdot \sin\theta)^{0.65}$$

where:

K_2	= Correction factor for angle of attack of flow
θ	= Angle of attack of the flow with respect to the pier
L	= Length of pier, ft (m)
a	= Pier width, ft (m)

The maximum value of L/a to be used in this equation is 12.

The correction factor for bed condition, K_3, can be obtained from table 8.2.2.7.

Table 8.2.2.7 Correction factor for bed condition

Bed Condition	Dune Height (ft)	Dune Height (m)	K_3
Clear-water scour	n/a	n/a	1.1
Plane bed and antidune flow	n/a	n/a	1.1
Small dunes	12 > H ≥ 2	3 > H ≥ 0.6	1.1
Medium dunes	30 > H ≥ 10	9 > H ≥ 3	1.2 to 1.1
Large dunes	H ≥ 30	H ≥ 9	1.3

The correction factor for armoring, K_4, can be determined using the following equations:

$$K_4 = \left[1 - 0.89 \cdot (1 - V_R)^2\right]^{0.5}$$

$$V_R = \left[\frac{V_1 - V_i}{V_{c90} - V_i}\right]$$

$$V_i = 0.645 \cdot \left[\frac{D_{50}}{a}\right]^{0.053} \cdot V_{c50}$$

where:

V_R = Velocity ratio
V_1 = Approach velocity, ft/s (m/s)
V_i = Approach velocity at which particles begin to move, ft/s (m/s)
V_{c90} = Critical velocity for D_{90} bed material size, ft/s (m/s)
V_{c50} = Critical velocity for D_{50} bed material size, ft/s (m/s)
a = Pier width, ft (m)

$$Vc = C \cdot y^{1/6} \cdot D_c^{1/3}$$

C = constant = 10.95 (6.19 for metric units)
y = depth of water just upstream of pier, ft (m)
D_c = Critical particle size for the critical velocity V_c, ft (m)

The minimum median bed material size, D_{50}, for computing K_4 is 0.2 ft (0.06 m). The minimum value of K_4 is 0.7. V_R must be greater than or equal to 1.0.

The pier width used in the above equations is that projected normal to the direction of flow. Piers should be skewed to minimize this width. The effect of debris should be considered in evaluating pier scour by considering the width of accumulated debris in determining the pier width used in the above equations.

For multiple columns with a spacing of 5 diameters or more, the total pier scour is limited to a maximum of 1.2 times the scour depth calculated for a single column. For multiple columns spaced less than 5 diameters apart, a "composite" pier width that is the total projected width normal to the angle of attack of flow should be used.

For example, for three 6 ft piers spaced at 30 ft apart, the pier width is somewhere between 6 ft and 18 ft (three times six feet), depending on the angle of attack.

Top width of pier scour holes, measured from the pier to the outer edge of the scour hole, can be estimated as 2.0 y_s.

Abutment scour - Two equations are available for determining scour at abutments. The first equation, by Froelich, is given as:

$$y_s = 2.27 \cdot K_1 \cdot K_2 \cdot (L')^{0.43} \cdot Y_a^{0.57} \cdot F_r^{0.61} + y_a$$

where:

y_s	= Scour depth, ft (m)
K_1	= Coefficient for abutment shape
K_2	= Coefficient for angle of embankment to flow = $(\theta/90)^{0.13}$
θ	= Angle between embankment and flow (degrees), $\theta >$ 90 if embankment points upstream
L'	= Length of embankment projected normal to flow, ft (m)
y_a	= Average depth of flow on the floodplain, ft (m)
Fr	= Froude number of approach flow upstream of abutment
	= $V_e/(g \cdot y_a)^{1/2}$
V_e	= Q_e/A_e, ft/s (m/s)
Q_e	= flow obstructed by abutment and approach embankment, ft^3/s (m^3/s)

The second equation, from the FHWA publication Highways in the River Environment (HIRE), is recommended when the ratio of projected abutment length, L', to flow depth, y_a, is greater than 25. This equation is given as:

$$y_s = 7.27 \cdot y_1 \cdot K_1 \cdot K_2 \cdot Fr^{0.33}$$

where:

y_s	= Scour depth, ft (m)
y_1	= Flow depth at the abutment, ft (m)
K_1	= Coefficient for abutment shape
K_2	= Coefficient for angle of embankment
Fr	= Froude number of approach flow upstream of abutment

The value of K_1 in both of the above equations can be obtained from Table 8.2.2.8.

Table 8.2.2.8 Coefficient for abutment shape

Abutment shape	K_1
Vertical-wall abutment	1
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

The value of K_2 in the HIRE equation can be obtained from Figure 16 in HEC-18 (13).

Total scour

All the above types of scour are considered in determining proper depth of bridge foundations. The total scour is obtained by adding the individual scour components.

Provide justification if scour analysis is not performed (slope protection can eliminate the need for abutment scour calculations, etc.)

8.2.2.7 Engineering Evaluation of Selected Alternatives

Evaluation of the consequences of risk associated with a stream crossing considers capital cost, traffic service, environmental and property impacts and hazards to human life. The evaluation of risk is a two-stage process. The initial step, identified as risk assessment, is a qualitative analysis of the potential risk involved in the stream crossing. The risk assessment should consider damage to structures, embankments, and surrounding property, traffic related losses, and scour or stream channel change. In each of the following categories, the risk is compared to a threshold value:

- lack of a practicable detour
- hazard to people
- hazard to property

If the threshold is exceeded for any one of the categories, the second stage of the risk analysis process, the Least Total Economic Cost (LTEC) design, should be employed. The FHWA publication HEC-17 (14) provides detailed procedures for performing a LTEC design.

8.2.2.8 Documentation of Hydraulic Design

Documentation is viewed as the record of reasonable and prudent design analysis based on the best available technology. Documentation should be an on-going process throughout the design and life of the structure.

Proper documentation achieves the following:

- protects MoDOT and the designer by proving that reasonable and prudent practices were used (be careful to state uncertainties in less than specific terms)
- identifies site conditions at time of design
- documents that practices used were commensurate with the perceived site importance and flood hazard
- provides continuous site history to facilitate future construction

At a minimum, the following documentation of the hydraulic design is to be archived:

- Bridge Survey Report form, associated plan and profile sheets
- Bridge Hydraulics and Scour Report or Culvert Hydraulics Report
- Any computation sheets used in the hydrologic and hydraulic analyses
- Input/output files from water surface profile model(s), HY-8, and other computer programs. If input files are not provided, all input must be reproducible from the information provided in the output.

8.2.3 National Flood Insurance Program

8.2.3.1 Floodplain Development Permit

Communities (cities, counties or states) participating in the National Flood Insurance Program (NFIP) are required to regulate construction in areas subject to potential flooding. The community regulates construction by requiring permits for development in special flood hazard areas. The State Emergency Management Agency (SEMA) has been granted authority to regulate floodplain development by state agencies and to issue floodplain development permits for state projects. SEMA requires a floodplain development permit for any development in special flood hazard areas, regardless of whether the community is participating in the NFIP.

The Structural Project Manager must obtain the necessary floodplain development permit(s) from SEMA for construction in a regulated special flood hazard area.

8.2.3.2 Floodplain and Special Flood Hazard Area

A floodplain is defined by the Federal Emergency Management Agency (FEMA) as any land area susceptible to being inundated by water. The 100-year flood, or a flood with a one percent annual chance of being equaled or exceeded, has been adopted by FEMA as the base flood for the NFIP. The water surface elevation of the base flood is known as the base flood elevation (BFE). A special flood hazard area is land in the floodplain inundated by the 100-year flood and is commonly referred to as the "100-year floodplain." A floodplain development permit is required for any construction in a special flood hazard area. Special flood hazard areas are typically shown as "A zones" on flood insurance maps.

8.2.3.3 Floodway

Encroachment on the floodplain, such as roadway fill, reduces the flood-carrying capacity, increases the flood heights of streams and increases flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For the purposes of the NFIP, the floodway concept is used as a tool to assist in this aspect of floodplain management. The 100-year floodplain is divided into a floodway and a floodway fringe. The floodway is the channel of the stream plus the portions of the adjacent overbanks which must be kept free of encroachment in order to pass the base flood without cumulatively increasing the water surface elevations by more than a designated height. The floodway fringe is the area between the floodway and floodplain boundaries (see Figure 8.2.3.1).

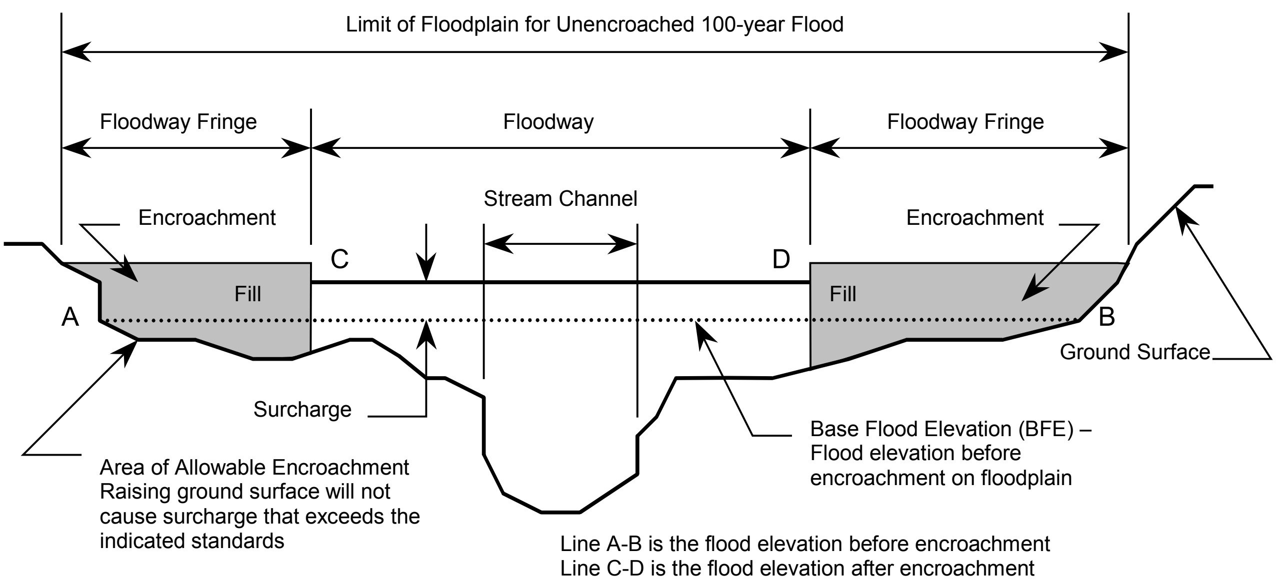

Figure 8.2.3.1. Floodplain Encroachment and Floodway

Construction within a floodway

Construction in the floodway that causes any increase in the BFE is prohibited. In order to issue a floodplain development permit for construction in the floodway, the community must receive a "No-Rise Certification" provided by a registered professional engineer, which certifies that the proposed construction will cause no increase in the BFE.

Several methods can be used to demonstrate that a construction project within a floodway will not cause an increase in the BFE. The simplest method is to model both the existing conditions and the proposed conditions. Comparison of the water surface elevations from these two models will show any increase caused by the construction; generally, if the project as a whole causes no increase in the BFE, that portion within the floodway will also cause no increase.

Another method is to include only that portion of the project within the floodway in a "proposed conditions" model. Comparison of these water surface elevations to the existing conditions water surface elevations will directly show the impact of the proposed construction in the floodway.

It is generally not difficult to show no increase in BFE's for bridge replacements where the existing bridge is on or near the existing alignment; new bridges are usually longer and cause less obstruction to the 100-year discharge than existing bridges. For bridges on new alignment, additional steps must sometimes be taken to cause no increase in BFE's. Possibilities include modification of the roughness coefficients through the structure or excavation of material from the overbanks for some distance upstream and downstream of the structure. All such modifications must be justifiable.

Temporary Bridges
Temporary bridges designed to pass the 10-year discharge with 1.0 foot (0.3 m) of backwater will typically result in an increase in base flood elevations. Permits for temporary bridges in floodways will be handled by SEMA on a case-by-case basis. The floodplain development permit application for temporary bridges must include the following:

- a hydraulic analysis of the effect of the temporary bridge on base flood elevations,
- a determination of the effect of any increased flooding resulting from the temporary bridge on any upstream improvements,
- and, an estimate of length of time temporary bridge will be in place

Culvert Extensions
Culvert extensions in floodways can pose a particularly challenging problem depending on whether they operate under inlet control or outlet control.

Culverts operating under inlet control can generally be lengthened without increasing water surface elevations. In some cases, an improvement to the inlet may be required to compensate for increases in culvert length.

Culverts operating under outlet control generally can not be lengthened without increasing water surface elevations upstream.

Floodway Revisions

Where construction in an existing floodway is absolutely necessary, and such construction will cause an increase in the BFE, the flood insurance study or floodway must be revised so that the proposed construction no longer causes an increase in the BFE or is no longer in the floodway. Flood insurance study revisions are obtained from FEMA through the community or communities with jurisdiction. The revision process requires a detailed hydraulic analysis and the cooperation and approval of all communities involved.

In general, obtaining a revision is a difficult and time-consuming process and should be avoided if at all possible. However, revising the floodway can be particularly cost-effective in one situation. Floodway widths are determined precisely only at the locations of cross-sections in the hydraulic model used to create the FIS. At all other locations along the stream, floodway widths are determined by interpolation along topographic maps. When a stream crossing is located between cross-sections, at a significant distance from both the upstream and downstream cross-section, it may be beneficial to review the hydraulic model used in the FIS. In some cases, adding an additional cross-section to the model at the location of the proposed structure will allow the floodway width to be reduced at that location, especially if the floodway appears unusually wide at the structure location.

8.2.3.4 Review of Flood Insurance Study and Maps

The Bridge Division maintains in its files copies of the FEMA Flood Insurance Study (FIS) reports and associated maps for streams subject to the National Flood Insurance Program.

Community Status Book

A current list of communities for which flood insurance studies have been performed is available in the Community Status Book (CSB), published on the Internet at :

http://www.fema.gov/CSB/mo.pdf

This list should be consulted to determine if a flood insurance study has been performed for any community within the project limits. The CSB list is divided into two parts: communities participating in the NFIP and communities that are not participating. Both parts of the list must be reviewed, as permits are required by SEMA for projects in a special flood hazard area when a flood insurance study has been performed, regardless of whether the community participates in the NFIP.

The CSB also includes the effective date of the current flood insurance study for the community. It is important to compare this date with the effective date of the FIS and maps in the Bridge Division files; if the CSB shows a later date, a revised study report and maps must be obtained. In rare instances, a flood insurance study may have been performed, but the study or map does not exist in the Bridge Division files. Copies of those documents can generally be obtained from SEMA.

Flood Insurance Study Reports

The study report contains valuable information regarding discharges, floodway widths, water surface elevations, and other items that may be pertinent to hydraulic design. Depending on the degree of flood hazard posed, a particular stream may have been analyzed by approximate methods or by detailed hydrologic and hydraulic methods. The level of information presented in the study can vary greatly depending on whether the stream in question was studied by detailed or approximate methods. The report for any communities within the project limits should be carefully reviewed.

Flood Insurance Study Maps

The FIS maps may be one of three types: Flood Insurance Rate Maps (FIRMs), Flood Boundary and Floodway Maps (FBFMs), or Flood Hazard Boundary Maps (FHBMs). FHBMs are used when detailed studies have not been performed, no floodway has been developed, and floodplain boundaries are approximate. FIRMs and FBFMs are used when a detailed study has been performed and a floodway has been developed and show the boundaries of both the floodplain and the floodway. Special flood hazard areas are typically shown as Zone A on FHBMs and as Zone A, Zone AE, or Zones A1 through A30 on FIRMs and FBFMs.

Originally, FBFMs were used to delineate the floodway and FIRMs were used to delineate the various insurance rating zones. Recently, however, the two were combined, and now only the FIRM is published. The newer FIRMs delineate both rating zones and floodways. Depending on the publication date of the flood insurance study, it may be necessary to look at either a FBFM or a FIRM to determine whether the project lies within a regulatory floodway.

For all communities for which a flood insurance study has been performed, the maps that include a portion of the project should be checked to determine if the project is within a special flood hazard area. If so, a floodplain development permit is required.

If any portion of the project is to be constructed within a regulatory floodway, the portion of the construction within the floodway can not cause an increase in the BFE and a No-Rise Certification will be required by SEMA.

Summary of FIS Review Process

The process for reviewing floodway maps is summarized below:

- Check all communities within project limits to see if a flood insurance study has been performed.
- If study exists, check maps (FIRMs, FBFMs, FHBMs).
- If in special flood hazard area, floodplain development permit is required.
- If in regulatory floodway, can cause no increase in BFE. No-Rise Certification is required.
- If it is not possible to achieve no increase in BFE, a flood insurance study or floodway revision may be required.

8.2.3.5 Floodplain Development Permit Application and No-rise Certification

Ami Pro versions of SEMA's floodplain development and a sample No-Rise Certification are available (see page 3.5-2). The No-Rise Certification is to be signed by the Division Engineer, Bridge.

In filling out the floodplain development permit application, the following areas warrant particular care.

- determination of the quarter-quarter section, township and range,
- floodway/floodway fringe designation
- 100-year flood elevation – the FIS base flood elevation should be given if available,
- and, the current map date – check the community status book.

The project description must include all aspects of the proposed construction, including grading, fill, and pavement in addition to the proposed bridge.

A photocopy of the section of the relevant FIS map showing the project location, along with the map panel number, shall be included with the floodplain development permit application.

STATE OF MISSOURI
FLOODPLAIN DEVELOPMENT PERMIT/APPLICATION

Application # ____________________ Date: ____________________

TO THE ADMINISTRATOR: The undersigned hereby makes application for a Permit to develop in a floodplain. The work to be performed, including flood protection works, is as described below and in attachments hereto. The undersigned agrees that all such work shall be done in accordance with the requirements of the Executive Order and all other laws and regulations of the State of Missouri.

State Agency	Date	Builder	Date
Address		Address	
Phone		Phone	

SITE DATA

1. Location: ______ 1/4; ______ 1/4; Section ______ ; Township ______ ; Range ______
   Street Address ____________________
2. Type of Development: Filling ______ Grading ______ Excavation ______ Min Improvement ______
   Routine Maintance ______ Substantial Improv ______ New Const ______ Other ______
3. Description of Development: ____________________
4. Premises: Structure size ______ ft x ______ ft Area of site ______ sq. ft
   Principal use ______ Accessory uses (storage, parking, etc.) ______
5. Value of Improvement (fair market) $ ______ Pre-Improv./Assessed value of structure $ ______
6. Property located in a designated FLOODWAY? Yes ______ No ______
   IF ANSWERED YES, CERTIFICATION MUST BE PROVIDED PRIOR TO THE ISSUANCE OF A PERMIT TO DEVELOP, THAT THE PROPOSED DEVELOPMENT WILL RESULT IN NO INCREASE IN THE BASE FLOOD (100-YEAR) ELEVATION.
7. Property located in a designated floodplain FRINGE? Yes **(Zone "?")** No ______
8. Elevation of the 100-year flood (ID source) ____________________ MSL/NGVD
9. Elevation of proposed development site ____________________ MSL/NGVD
10. Elevation/floodproofing requirement ____________________ MSL/NGVD
11. Other floodplain elevation information (ID and describe source)
    ____
12. Other permits required? Corps of Engineer 404 Permit: Yes ______ No ______ Provided ______
    State Dept. of Natural Resources: Yes ______ No ______ Provided ______

______ All provisions of Executive Order 97-09, Floodplain Management Executive Order shall be in compliance.

PERMIT APPROVAL/DENIAL

Plans and Specifications Approved/Denied this ______ Day of ____________________ , ______

Signature of State Agency	Authorizing Official
Print Name and Title	Print Name and Title

THIS PERMIT ISSUED WITH THE CONDITION THAT THE LOWEST FLOOR (INCLUDING BASEMENT FLOOR) OF ANY NEW OR SUBSTANTIALLY-IMPROVED RESIDENTIAL BUILDING WILL BE ELEVATED ______ METER(S) ABOVE THE BASE FLOOD ELEVATION. IF THE PROPOSED DEVELOPMENT IS A NON-RESIDENTIAL BUILDING, THIS PERMIT IS ISSUED WITH THE CONDITION THAT THE LOWEST FLOOR (INCLUDING BASEMENT) OF A NEW OR SUBSTANTIALLY-IMPROVED NON-RESIDENTIAL BUILDING WILL BE ELEVATED OR FLOODPROOFED ______ METER(S) ABOVE THE BASE FLOOD ELEVATION.

THIS PERMIT IS USED WITH THE CONDITION THAT THE STATE AGENCY WILL PROVIDE CERTIFICATION BY A REGISTERED ENGINEER, ARCHITECT, OR LAND SURVEYOR OF THE "AS-BUILT" LOWEST FLOOR (INCLUDING BASEMENT) ELEVATION OF ANY NEW OR SUBSTANTIALLY-IMPROVED BUILDING COVERED BY THIS PERMIT.

8.2.4 Legal Aspects of Hydraulic Design

8.2.4.1 Overview

An understanding of drainage law is essential to the responsible design of highway drainage facilities. In general, the following statements can be made about the legal aspects of hydraulic design:

- Natural drainage should be perpetuated as far as possible.
- Infliction of damage that could reasonably have been avoided is looked upon with disfavor by courts.
- Drainage law standards are becoming more flexible and depending more on the circumstances of each particular case.

The designer is advised to consult with the Chief Counsel's Office on any matters involving potential legal liability or litigation.

8.2.4.2 Federal Laws

The State is required to comply with all Federal Laws. The Code of Federal Regulations (CFR) includes all regulations in force at time of publication. The following federal laws significantly affect highway drainage design:

- National Environmental Policy Act of 1969 (NEPA) (42 USC 4321-4347) (23 CFR 771)
- National Flood Insurance Program (44 CFR 59-77)
- Navigable Waters (Section 404 Permits, Section 401 Water Quality Certification, etc.) (33 USC 1344)
- Fish and Wildlife regulations (endangered species, etc.)

8.2.4.3 State Laws

Reasonable Use Rule

State law regarding surface waters and flood waters generally follows the *Reasonable Use Rule*, stated as follows:

Possessor is privileged to make reasonable use of his land even though alteration of flow of surface waters causes harm to others. Liability is incurred only when interference with flow of surface waters is deemed unreasonable.

The definition of reasonable is unclear, and will vary on a case by case basis. However, the test for reasonableness should consider such items as the amount of harm caused, the foreseeability of harm and the purpose or motive of altering surface flows.

Stream Water Rules

In addition to the reasonable use rule, interference with the flow of a natural watercourse causing damage to another party or diversion of flow from one watercourse to another will generally result in liability.

Eminent Domain/Inverse Condemnation

A structure that impacts either surface waters or stream waters is likely to result in liability to the adversely affected landowner. If the right to do so is not acquired by deed or condemnation, the landowner may institute an inverse condemnation suit, which, if successful, will result in award of damages and attorney's fees.

8.2.4.4 Local Laws

Generally, the State is not required to comply with local ordinances, except where compliance is required by State statute. The State may choose to comply as a matter of courtesy if no burden is imposed on the State.

8.2.4.5 Common Drainage Complaints

Listed below are several common causes for drainage complaints by landowners. Consideration should be given to minimizing or eliminating, to the extent practicable, these causes for complaint:

- Diversion of flow from one watercourse to another
- Collection and concentration of surface waters
- Augmentation of flow peaks or volumes
- Obstruction of flows resulting in increased backwater
- Erosion and sedimentation
- Groundwater interference

8.2.4.6 Significant Court Decisions

Hines Implement - Court decision imposes liability for upstream water damage caused by diverting surface water if use is deemed unreasonable. Reasonableness is determined on a case by case basis. This case represents the first use of the Reasonable Use Rule in a decision involving MoDOT.

The department can be held liable for downstream impacts. This has not been affected by the Hines Implement case.

8.2.4.7 Negligence and Liability

No hydraulic design is without risk, and some degree of risk must be accepted in the final design. However, damages that were not anticipated in the design may be viewed as due to negligence. It is appropriate to have foreseen the possibility of damage, weighed it against other factors, and accepted that risk as a proper exercise of discretionary judgment. Use of sound engineering judgment, accepted design procedures and sufficient documentation is essential, and provides a defense against liability due to negligence.

In theory, an engineer may be held personally liable for negligent design; however such suits have never been filed to date against a MoDOT engineer. If a suit is filed and the engineer cooperates with the CCO, the engineer will be provided a complete defense and MHTC will pay any damages awarded.

8.2.5 Hydraulic Design References

8.2.5.1 List of References

1. Stream Stability at Highway Structures*, Federal Highway Administration Publication FHWA-IP-90-014 - Hydraulic Engineering Circular No. 20, November 1995*
2. *AASHTO Highway Drainage Guidelines, American Association of State Highway and Transportation Officials, 1992*
3. Guidelines for Determining Flood Flow Frequency*, United States Water Resources Council, Bulletin #17B of the Hydrology Committee, September 1981*
4. Technique for Estimating the 2- to 500-Year Flood Discharges on Unregulated Streams in Rural Missouri*, Alexander and Wilson, USGS Water-Resources Investigations Report 95-4231, 1995*
5. Techniques for Estimating Flood-Peak Discharges for Urban Basins in Missouri*, Becker, USGS Water-Resources Investigations Report 86-4322, 1986*
6. HEC-RAS User Manual*, US Army Corps of Engineers, April 1997*
7. HEC-RAS Hydraulic Reference Manual*, US Army Corps of Engineers, April 1997*
8. HEC-RAS Applications Guide*, US Army Corps of Engineers, April 1997*
9. Roughness Characteristics of Natural Channels*, Barnes, USGS Water-Supply Paper 1849, 1977*
10. Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains*, Federal Highway Administration, Report No. FHWA-TS-84-204, April 1984*
11. Open-Channel Hydraulics*, Ven Te Chow, McGraw Hill Book Company, 1988, pp. 108-123*
12. Hydraulic Design Series No. 5 - Hydraulic Design of Highway Culverts*, Federal Highway Administration, Report No. FHWA-IP-85-15, September 1985*
13. Evaluating Scour at Bridges*, Federal Highway Administration Publication FHWA-IP-90-017 - Hydraulic Engineering Circular No. 18, November 1995*
14. The Design of Encroachments on Flood Plains Using Risk Analysis*, Federal Highway Administration - Hydraulic Engineering Circular No. 17, April 1981*